Bridge Decks with Precast UHPC Waffle Panels: A Field Evaluation and Design Optimization

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Abstract
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Keywords
Precast, Waffle panel, Ultrahigh-performance concrete (UHPC), Accelerated bridge construction (ABC), Bridge deck, Optimization, Design

Disciplines
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Comments

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BRIDGE DECKS WITH PRECAST UHPC WAFFLE PANELS: A FIELD EVALUATION AND DESIGN OPTIMIZATION

Ebadollah Honarvar¹, Sri Sritharan², Jon Matthews Rouse³, and Sriram Aaleti⁴

ABSTRACT

The first full-depth, precast, ultra-high performance concrete (UHPC) waffle panels have been designed and implemented in a bridge replacement project to utilize accelerated bridge construction (ABC) and increase bridge deck longevity. This paper first evaluates the structural performance of this bridge using a combination of field live load testing and analytical modeling. The collected data for vertical deflections and strains at discrete, critical locations on the bridge deck, subjected to static and dynamic truck loads, demonstrated satisfactory performance of the bridge deck and correlated well with the results obtained from the analytical model. Thereupon, options to optimize the bridge deck are examined to minimize the UHPC volume and associated labor costs. Using the analytical model, an optimization of the waffle panels was undertaken by varying the number of ribs as well as spacing between the ribs. An optimized panel was achieved by reducing the interior ribs per panel from four to two, or zero, in the longitudinal direction and six to two in the transverse direction, without compromising the panel’s structural performance.

Author keywords: Precast; Waffle panel; Ultra-high performance concrete (UHPC);
Accelerated bridge construction (ABC); Bridge deck; Optimization; Design

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Introduction

Current bridge infrastructure challenges in the U.S. caused by growing traffic volume and an increasing number of aging, structurally deficient or obsolete bridges, demand accelerated bridge construction (ABC) methods and structural systems with increased longevity. The Better Roads Bridge Inventory survey (2009) indicated that the deterioration of the deck is a leading cause for obsolete and/or a deficient inspection rating of the bridges. Due to the excellent durability and structural properties of ultra-high performance concrete (UHPC), it has been receiving more attention by bridge engineers as a means to increase the bridge service life and reduce life-cycle costs by requiring less maintenance (Piotrowski and Schmidt 2012).

The dense matrix of UHPC leads to enhance durability properties over the conventional concrete as measured by freeze-thaw tests, scaling tests, permeability tests, resistance to alkali-silica reactivity (ASR), abrasion tests, and carbonation (Russell and Graybeal 2013). Hence, the use of UHPC in bridge deck application prevents the detrimental solutions from infiltrating into the matrix when it is designed to be crack free and exposed to the environmental deterioration.

However, currently the UHPC’s initial unit quantity cost far surpasses that of conventional concrete, which underscores the need for economy in its use, by optimizing the design as emphasized by the FHWA-HRT-13-060 report (Russell and Graybeal 2013). Additionally, utilizing precast concrete deck panels is gaining significant interest among several State Departments of Transportation (DOTs) for both new and replacement bridges, as a system promoting ABC (Terry et al. 2009). Previously, Issa and Yousif (2000) and Berger (1983) showed that the use of precast, full-depth concrete deck systems can significantly accelerate
bridge construction/rehabilitation, resulting in minimized delays and disruptions to the community.

For the reasons noted above, the State of Iowa, which has the third highest number of deficient bridges in the U.S. (ASCE 2013), has been actively implementing UHPC in its infrastructure. The Iowa DOT led the nation with the implementation of UHPC Pi girders (Keierleber et al. 2008) and the development of an H-shaped UHPC precast pile for foundation applications (Vande Voort et al. 2008). In one of the recent projects sponsored by the FHWA Highways for LIFE (HfL), by combining the advantages of UHPC with those of precast deck systems, a bridge system with prefabricated UHPC waffle deck panels and field-cast UHPC connections was developed. Following a successful laboratory evaluation of the structural performance of waffle deck panels and suitable connections (Aaleti et al. 2011), a full-scale, 19.2 m (63 ft) long, single span demonstration bridge with full depth prefabricated UHPC waffle deck panels was constructed on Dahlonega Road in Wapello County, Iowa. This replacement bridge is the first UHPC waffle deck bridge in the U.S. and is used to demonstrate the deployment of the UHPC waffle deck technology from fabrication through construction.

In the first part of this paper, field testing in conjunction with an analytical study using 3D finite element analysis (FEA) software, ABAQUS was completed to evaluate the structural performance of the Dahlonega Road Bridge deck. The field testing conducted as a part of this study included monitoring live load vertical deflections and strains at discrete, critical locations on the bridge superstructure, as it was subjected to static and dynamic truck loads. A preliminary finite element model (FEM) of the bridge was developed using ABAQUS and validated to help interpret the results of live load testing, estimate strains due to dead load, and to examine live load moment distribution.
With an intention of reducing UHPC volume and the waffle deck cost, the second part of this paper investigates cost effective design alternatives to the deck design completed for Dahlonega Road Bridge. An optimization of the waffle panels was undertaken by varying the number of ribs as well as the spacing between the ribs, using the FEM, reducing the UHPC volume by as much as 13.4%. The design guidelines proposed for the implementation of UHPC waffle deck systems in new and replacement bridges, by Aaleti et al. 2013, were given consideration in the optimization study. Furthermore, girder live load moment distribution factors (DFs) of the optimized designs were calculated and compared with the current design to ensure that the optimal designs would not alter the distribution of loads between the girders and that the bridge superstructure would act effectively as an integral system.

**Bridge Description**

The single-span, two-lane Dahlonega Road Bridge, the replacement of an existing bridge in Wapello County, Iowa, is 9.14 m (33 ft) wide and 19.20 m (63 ft) long. It consists of fourteen prefabricated, full-depth, precast concrete panels installed on five standard Iowa “B” girders (Index of Beam Standards 2011) placed at a center-to-center distance of 2.33 m (7 ft and 4 in.). The bridge plan view, cross section, and construction photos are shown in Fig. 1.

A single UHPC waffle panel of the Dahlonega Road Bridge deck is 5.5 m (16 ft and 2.5 in.) wide and 2.44 m (8 ft) long, as shown in Fig. 2a. Note that the terms, longitudinal and transverse used throughout this document are relative to the bridge, not the panel. Each of the two cells in a panel have three interior ribs and two interior ribs in the transverse and longitudinal directions, respectively, and two exterior ribs in each direction, as illustrated in Fig. 2b and Fig. 2c. Hereafter, the interior ribs in each cell of a panel are referred to simply as ribs. Each rib is 101
mm (4 in.) wide at the top with a gradual decrease to 76 mm (3 in.) at the bottom, and 140 mm (5.5 in.) deep. Longitudinal and transverse ribs were both reinforced with No.19 (No.6, \(d_b = 0.75\) in., \(d_b\) is diameter of bar) bars at the top and the bottom. Stainless steel dowels with a diameter of 25 mm (1 in.) were used to reinforce the field-cast UHPC joints. The panels were connected across the length of the bridge using a transverse joint connection. In this connection, panel’s dowel bars were tied together with additional transverse reinforcement and the gap between the panels was filled with UHPC, as exhibited in Fig. 2d. In order to make the girders fully composite with the panels, a shear pocket connection and a waffle panel-to-girder longitudinal connection were provided, as shown in Fig. 3. In addition, the performance of composite connection between UHPC deck panels and girders was evaluated to be satisfactory by Graybeal (2014). More details for panel reinforcement and connections can be found in the report by Aaleti et al. (2013).

Fig. 1. Dahlonega Road Bridge: (a) plan view; (b) cross section; (c) construction
Fig. 2. Single UHPC waffle panel: (a) plan view; (b) longitudinal cross section A-A; (c) transverse cross section B-B; (d) panel to panel connection (Aaleti and Sritharan 2014)
Field Testing

To ensure a satisfactory response of the panels under true service conditions, two UHPC waffle deck panels, next to the east barrier, were selected for instrumentation, as shown in Fig. 1. Each of the two panels, one located near the mid-span and the other one located adjacent to the south abutment, was instrumented with the surface mounted BDI strain transducers to quantify deformations and identify the likelihood of cracking under live load. Each transducer was labelled based on its location and orientation. The nomenclature for transducers and the location of transverse and longitudinal grid lines are presented in Tables 1 and 2, respectively.

At the mid-span panel, eight transducers were placed on the bottom of the deck in maximum positive moment regions, and seven were placed on the top of the deck at regions of maximum
negative moment, as shown in Fig. 4. Of the total 15 transducers, seven transducers, located on the UHPC infill deck joint and the interface between the joint and panel, were used to identify distress in the joint regions or the opening of the interface between joint and panel. At the panel adjacent to the abutment, six strain transducers were placed on the bottom of the deck at regions of maximum positive moment, and four were placed on the top of the deck in regions of maximum negative moment, as shown in Fig. 5. Of these ten transducers, two were located to span the interface between the UHPC infill joint and UHPC precast panel in order to identify an opening at this interface.

<table>
<thead>
<tr>
<th>Table 1- Transducers Nomenclature</th>
</tr>
</thead>
<tbody>
<tr>
<td>First character</td>
</tr>
<tr>
<td>Span Location</td>
</tr>
<tr>
<td>M: Mid-Span</td>
</tr>
<tr>
<td>A: Near Abutment</td>
</tr>
</tbody>
</table>

*See bridge plan (Fig. 1), and Table 2 for grid locations. Example: MDTT13 corresponds to mid-span deck panel, oriented transversely on top along longitudinal Grid Line 1 and transverse Grid Line 3

<table>
<thead>
<tr>
<th>Table 2- Location of Transverse and Longitudinal Grid Lines</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transverse grid line</td>
</tr>
<tr>
<td>----------------------</td>
</tr>
<tr>
<td>0</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>4</td>
</tr>
<tr>
<td>5</td>
</tr>
<tr>
<td>-</td>
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</tbody>
</table>

In addition to the strain transducers on the deck panels, 13 strain transducers and five string potentiometers were attached to the girders to characterize the global bridge behavior, measure mid-span deflections, and quantify lateral live load moment distribution factors (see Fig. 4 and
Fig. 5). Top and bottom girder strains were monitored for three of the girders at mid-span and at a section 0.67 m (2.21 ft) from the south abutment.

Fig. 4. Panel at mid-span: (a) location of transducers on top and bottom of deck; (b) cross section view A-A

Fig. 5. Panel near abutment: (a) location of transducers on top and bottom of deck; (b) cross section view A-A
Loading

Live load was applied by driving a loaded dump truck across the bridge, along seven predetermined paths, as shown in Fig. 6a. Load paths one and seven were 0.61 m (2 ft) from each barrier rail for the outer edge of the truck. Load paths two and six were along the centerline of each respective traffic lane. Load paths four and five were 0.61 m (2 ft) to either side of the bridge centerline for the outer edge of the truck, and load path three straddled the centerline of the bridge.

The total weight of the truck was 27,306 kg (60,200 lbs.) in accordance with the guide for the field testing of bridges, by Working Committee on the Safety of Bridges (1980), with a front axle weight of 8,233 kg (18,150 lbs.), and two rear axles weighing roughly 9,525 kg (21,000 lbs.), each. The truck configuration with axle loads is shown in Fig. 6b.

For static tests, the truck was driven across the bridge at a crawl [speed < 2.25 m/s (5 mph)]. Each load path was traversed twice to ensure repeatability of the measured bridge response. For dynamic tests, the truck speed was increased to 13.4 m/s (30 mph) to examine dynamic amplification effects.

Fig. 6. Loading: (a) schematic layout of bridge loading paths; (b) truck configuration and axle load
Field Test Results

Because the load test captured only incremental live load deformations, the total strains were computed by superimposing the dead load strains computed with the FEM of the bridge, with the measured live load strains from the load test. For the deck panels, the dead load strains typically comprised only a minor portion of the total strains (i.e., less than 10%) because the waffle slab panels were significantly lighter than a conventional cast-in-place concrete deck. However, dead load strains comprised as high as 70% of the total strain for the precast girders. Because the predicted dead load strains were negligible for the deck, the presented results include maximum live load transverse and longitudinal strains of the mid-span panel and the panel near the abutment. Throughout this paper, negative values represent compressive strains and downward deflections, whereas positive values represent tensile strains and upward deflections.

The maximum transverse strains observed for each load path, with the corresponding transducer location at the mid-span panel and the panel near the south abutment, are presented in Fig. 7. All maximum strains for the UHPC waffle deck slab at the mid-span are significantly less than 250 µε, the cracking strain suggested for UHPC (Russell and Graybeal, 2013). This behavior implies that there was no cracking in this deck panel, and it was responding elastically to the applied truck load. Additionally, the values registered for gages MDTT35 and MDTT33 did not exhibit significantly high tensile strains (i.e., less than 20 µε), indicating good bonding between the precast panels and UHPC infill joints.

Unlike the mid-span panel, some hairline flexural cracks were observed on the bottom of the ribs on the panel adjacent to the south abutment, prior to loading, most likely caused at some point during storage, shipping, or erection. Consequently, relatively higher strains were observed at
these locations (e.g., gages ADTB2a2 and ADLB1a2) during the live load test when compared to strains in the mid-span panel. However, these strains are comparable to the expected cracking strain of the UHPC (250µε). Moreover, if the connection and proximity of the end panel to the abutment contributed to the elevated strains in this region, the strain recorded by gage ADTB2a1 would also be expected to register a similar strain level, which was not the case. However, since they are on the bottom of the deck and are not excessive in magnitude, small cracks at these locations are unlikely to pose a threat to the long-term performance of the panel.

In addition, the maximum longitudinal strains observed for each load path, with the corresponding transducer location at the mid-span panel, and the panel near the abutment, are presented in Fig. 8. The results indicate that maximum longitudinal strains are typically smaller and less critical than maximum transverse strains. Only at transducer ADTB1b2 for load path one did the panel exhibit high strains, which was due to the preexisting crack at the bottom of the panel near the abutment, as outlined previously.

**Fig. 7.** Maximum measured transverse strain for each load path with the corresponding transducer location: (a) mid-span panel; (b) panel near abutment
Fig. 8. Maximum measured longitudinal strain for each load path with the corresponding transducer location: (a) mid-span panel; (b) panel near abutment

**Girder Live Load Moment DF**

A DF is the fraction of the total load that a girder must be designed to sustain, when all lanes are loaded, to create the maximum effects on the girder. The distribution factor can be calculated from the load fractions based on displacements. Load fraction is defined as the fraction of the total load supported by each individual girder for a given load path. Thus, the load fractions for paths two and six (i.e., when the truck is located at the centerline of each respective lane) are calculated based on the displacement, as below:

\[
LF_i = \frac{d_i}{\sum_{i=1}^{n} d_i} \tag{1}
\]

where \(LF_i\) is load fraction of the \(i^{th}\) girder, \(d_i\) is deflection of the \(i^{th}\) girder, \(\sum d_i\) is the sum of all girder deflections, and \(n\) is number of girders.

Hence, the distribution factor for each girder can be computed as below:

\[
DF_i = LF_{2i} + LF_{6i} \tag{2}
\]

where \(DF_i\) is distribution factor of the \(i^{th}\) girder, \(LF_{2i}\) is load fraction from path 2 of the \(i^{th}\) girder, \(LF_{6i}\) is load fraction from path 6 of the \(i^{th}\) girder.
The maximum calculated DF was 0.51 and 0.38 for the interior and the exterior girders, respectively. Also, the DF values for interior and exterior girders were computed according to AASHTO LRFD Bridge Design Specification (2010). Case (k) from AASHTO LRFD Table 4.6.2.2.1-1, precast concrete I section with precast concrete deck is the most comparable to the Dahlonega Road Bridge system. Table 3 shows the results from AASHTO distribution factor equations as well as average distribution factors for interior and exterior girders, calculated using the measured vertical deflections.

Table 3- Live Load Moment DFs

<table>
<thead>
<tr>
<th>Girder</th>
<th>DF AASHTO</th>
<th>DF Displacement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interior</td>
<td>0.66</td>
<td>0.44±0.10</td>
</tr>
<tr>
<td>Exterior</td>
<td>0.63</td>
<td>0.34±0.06</td>
</tr>
</tbody>
</table>

The results indicate that AASHTO equations are conservative for both interior and exterior girders. This implies that the UHPC waffle deck has a higher stiffness than what is assumed in the 2010 AASHTO LRFD Bridge Design Specification.

**Dynamic Amplification Effects**

Dynamic tests were performed for load paths two, three, and six. The truck was driven at a speed of approximately 13.4 m/s (30 mi/h) along the bridge to quantify the dynamic amplification. The dynamic load allowance, also known as the DA, accounts for hammering effects due to irregularities in the bridge deck and resonant excitation, as a result of similar frequencies of vibration between bridge and roadway. The DA can be computed experimentally as follows:

\[
DA = \frac{\varepsilon_{\text{dyn}} - \varepsilon_{\text{stat}}}{\varepsilon_{\text{stat}}} \tag{3}
\]
where $\varepsilon_{dyn}$ is the maximum strain caused by the vehicle traveling at a normal speed at a given location, and $\varepsilon_{stat}$ is the maximum strain caused by the vehicle traveling at crawl speeds at the corresponding location. The dynamic amplification factor (DAF) is then given by:

$$\text{DAF} = 1 + DA$$  

(4)

Fig. 9a shows representative results for measured dynamic live load strains for load path three. Also, the calculated DAF of each girder for three different load paths are presented in Fig. 9b. The maximum DAF computed for the bridge girders is 1.41, which is slightly greater than the 1.33 recommended by AASHTO for design presumably, due to the relatively light weight of the waffle deck. Also, an investigation into the DA effect for transducers on the top of the deck revealed that some gages recorded relatively high DAFs, but none of the dynamic strains approached the assumed cracking strain for UHPC. Transducers on the bottom of the waffle deck panels also revealed some mild DA effects, but in all cases, the dynamic strains were well below those recorded in laboratory tests, reported by Aaleti et al. (2013).

**Analytical assessment**

A 3D nonlinear FEM was developed using ABAQUS software, Version 6-12. The geometric and reinforcement details were accurately employed in the FEM, as well as nonlinear material
properties. The waffle deck, girders, and abutments were modelled with deformable 8-node linear 3D stress elements (i.e., C3D8R in ABAQUS). The steel reinforcement in the deck panels and the abutments were modelled using two-node linear 3D truss elements (i.e., T3D2 in ABAQUS), with perfect bonding to the concrete. The integral abutments were modeled in accordance with the bridge design to impose a compatible movement of the superstructure (i.e., panels and girders) with the abutments.

The concrete in the prestressed girders and abutments was modelled using an elastic material with Young’s modulus of 32,874,000 kPa (4768 ksi), estimated using recommendations in AASHTO 2010. The UHPC behavior in the deck panels was represented with an inelastic material with the softening behavior, and was modelled using the Concrete Damaged Plasticity (CDP) model, available in ABAQUS. The stress-strain behavior of UHPC in tension and compression used in the FEA is shown in Fig. 10 (Aaleti et al. 2013). An idealized bilinear elastic plastic stress-strain material consecutive model was used to simulate mild steel reinforcement with Young’s modulus of 199,947,000 kPa (29000 ksi), a yield strength of 413,685 kPa (60 ksi), an ultimate stress of 620,527 kPa (90 ksi), and an ultimate strain of 0.12.

The load was applied in line with the truck configuration and load paths, as shown in Fig. 6. Each axle weight was equally distributed between two wheels located 2.44 m (8 ft) apart from each other. Then, the analysis was solved using the Static Riks solver in ABAQUS. Fig. 11 demonstrates the location of the truck for load path two with the corresponding deflected shape as representative of the entire performed analyses results.
Finite-Element Analysis Verification and Results

To assess the FEM’s accuracy in predicting the global bridge’s response to loads applied during the field test, calculated live load deflections and girder strains for load paths two and three were compared to the corresponding values measured during the test (see Tables 4 and 5, respectively). The calculated girder deflection and strain values presented in Tables 4 and 5 correspond to a critical truck location with the front axle of the truck placed at 16 m (52.5 ft) and 12.8 m (42 ft) from the south abutment for load path two and load path three, respectively.

From Table 4, it is clear that the finite element model accurately captured the maximum live load deflections for these two critical load paths for all of the girders. In most cases, the predicted
Deflections were within ±0.254 mm (±0.01 in.) of the measured values. As may be seen in Table 5 that the model is highly effective in predicting a strain response for the girders supporting the instrumented panels, where the discrepancy between the measured and estimated strain was within ten microstrain. These close comparisons of results obtained for the global response of the bridge provided confidence when examining the more local response of the waffle slab deck panels during the static load test.

**Table 4- Maximum Live Load Girder Deflections**

<table>
<thead>
<tr>
<th>Load Path</th>
<th>Deflection source (mm)</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>MGLB15</td>
</tr>
<tr>
<td>2</td>
<td>Test results</td>
<td>-0.818</td>
</tr>
<tr>
<td></td>
<td>FEM</td>
<td>-1.095</td>
</tr>
<tr>
<td>3</td>
<td>Test results</td>
<td>-0.180</td>
</tr>
<tr>
<td></td>
<td>FEM</td>
<td>-0.203</td>
</tr>
</tbody>
</table>

**Table 5- Girder top and bottom Longitudinal Strains at Mid-Span**

<table>
<thead>
<tr>
<th>Load Path</th>
<th>Strain source (με)</th>
<th>Location: Bottom</th>
<th>Location: Top</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>MGLB15</td>
<td>MGLB25</td>
</tr>
<tr>
<td>2</td>
<td>Test results</td>
<td>17</td>
<td>31</td>
</tr>
<tr>
<td></td>
<td>FEM</td>
<td>21</td>
<td>28</td>
</tr>
<tr>
<td>3</td>
<td>Test results</td>
<td>15</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>FEM</td>
<td>8</td>
<td>22</td>
</tr>
</tbody>
</table>

The maximum transverse strains at the locations of all transducers attached to the bottom of the mid-span panel and the panel near abutment were simulated in the FEM by placing the truck rear axle right above that transducer in line with the observed location during the field testing. The maximum estimated transverse strains of each transducer at the bottom of the mid-span panel and the panel near abutment are compared with those measured from the field test, as shown in Fig. 12 and Fig. 13, respectively. It can be observed that the FEM was able to estimate the strains for the mid-span panel accurately, where the highest deviation between the measured and the estimated strain was 38 microstrain. Contrarily, the predicted strains for the panel near abutments
were appreciably smaller than the measured strains due to preexisting cracks, as discussed previously in the field test results. In addition, the measured strain for the mid-span panel was compared to the calculated strain at discrete truck locations, along the bridge, to assess the reliability of FEM in capturing the strain distribution. Fig. 14 shows the results for the transverse strain at the critical locations at the bottom of the mid-span panel for load path two. It can be seen that the strain distribution is adequately captured by the FEM.

**Fig. 12.** Comparison of maximum transverse strains between field test and the FEM for the mid-span panel

**Fig. 13.** Comparison of maximum transverse strains between field test and the FEM for the panel near abutment
Optimization of Waffle Panels

The use of UHPC is limited in current day practice, partly due to high material costs, even though it exhibits superior structural characteristics, such as high compressive strength, reliable tensile strength, and improved durability. Therefore, for economical systems, an optimized design should be adopted to minimize the UHPC volume in structural members, without affecting the structural performance (Russell and Graybeal 2013). The newly developed design guide for the UHPC waffle deck (Aaleti et al. 2013) provides recommendations about the geometrical design of waffle panels, including, panel width, length, and thickness as well as rib dimensions and their spacing in transverse and longitudinal directions. Panel width and length are primarily governed by the bridge span and width, while the panel plate thickness is dictated by the punching shear capacity of the panel (Aaleti et al. 2013). The adequacy of punching shear capacity of 63.5 mm (2.5 in.) thick UHPC slab for bridge decks, subjected to AASHTO HL-93 truck [71.2 kN (16 kips) per tire] or Tandem truck [55.6 kN (12.5 kips) per tire] with the standard wheel load dimensions [254 mm (10 in.) by 508 mm (20 in.)], was validated. Aaleti et al. (2013) reported the measured average punching shear strength of 7,377 kPa (1.07 ksi), which
was nearly 2.3 times the estimated value using the equation recommended by Harris and Wollmann (2005). Therefore, both punching shear capacity values reported by Aaleti et al. (2013) and Harris and Wollmann (2005) are greater than the punching shear that would be experienced by a bridge deck when subjected to AASHTO truck.

In the context of minimizing the volume of UHPC for waffle deck panels, the number of ribs and ribs spacing can be potentially altered to reduce the UHPC volume. The remaining structural properties of components, such as panel dimensions and deck reinforcement, were retained during optimization.

In this study, two designs were investigated as alternatives to the waffle panel used in the Dahlonega Road Bridge, with the prospect of reducing the UHPC volume in line with the design guideline (Aaleti et al. 2013). The guideline recommends a maximum spacing of 0.91 m (36 in.) for the ribs in both longitudinal and transverse directions. However, these limits were slightly exceeded due to geometric constraints of the panel in the alternative designs.

The first alternative design reduced the number of ribs per cell, to one, in both longitudinal and transverse directions with a transverse and longitudinal rib spacing of 0.95 m (37.5 in.) and 1.05 m (41.5 in.), respectively. In the second alternative design, the longitudinal rib was eliminated as the load was primarily transferred in the transverse direction for the bridge deck. Therefore, the two longitudinal ribs in the original panel design were removed, while one transverse rib was retained. The elimination of the longitudinal ribs transformed the waffle slab effectively into the ribbed slab. It should be noted that the rib reinforcement [one continuous No. 19 (No. 6) reinforcing bar at the top and bottom of each rib] as well as rib tapering along the depth [101 mm (4 in.) wide at the top with a gradual decrease to 76 mm (3 in.) at the bottom] in the proposed designs were kept the same as the original design. Hereafter, the recommended designs are
referred to as redesign 1 (i.e., the design with one rib in both directions) and redesign 2 (i.e., the ribbed slab). Panel geometrical details for the original design, and redesigns one and two, are demonstrated in Fig. 15.

![Panel geometrical details for original design and redesigns.](image)

**Fig. 15.** Panel transverse and longitudinal cross sections juxtaposed with transverse strain results for load path 2 at the mid-span panel: (a) original design; (b) Redesign 1; (c) Redesign 2

The field test results indicated that peak strains in the deck panels occurred primarily for load path two (center of traffic lane) and load path three (straddling bridge centerline). Thus, evaluating the performance of the alternative designs, the analysis was conducted for these load paths. The location of the maximum transverse strain at the bottom of each panel for load path two is demonstrated in Fig. 15. The maximum estimated live load tensile strains at the bottom of
the panel for the three designs are reported in Fig. 16. It can be seen that the original design produced the smallest transverse strains, while redesign 2 produced the highest transverse strains. However, these strains are still lower than the UHPC cracking strain, thereby demonstrating satisfactory structural performance of the two proposed alternative designs. As expected, the longitudinal strains are fairly similar for the different designs. The strain distributions for the different designs at the critical location along the bottom of the mid-span panel were compared in Fig. 17. The results indicate that the proposed redesigns do not significantly change the strain distribution trend when compared to the original design and field measurements.

In the design guide (Aaleti et al. 2013), it was recommended to provide at least one interior longitudinal rib between two consecutive girder lines in addition to the exterior longitudinal ribs to ensure adequate connections between two adjacent panels. However, the load transfer in the current bridge seems to be in the transverse direction rather than the longitudinal direction. Hence, the adequacy of the connection between the two adjacent panels were analytically examined for redesign 2. As an extreme case, it was assumed that no bonding existed between the two adjacent panels except for the regions where there were exterior longitudinal ribs, which provided connectivity. The analysis showed that the maximum differential vertical deflection
between the two adjacent panels was 0.0002 m (0.01 in.), when the rear axle of the truck was placed in the mid-span panel. Consequently, the longitudinal rib can be removed without affecting the structural performance of the panels. The deflected shape of the two adjacent panels at the mid-span is illustrated in Fig. 18.

![Fig. 17. Comparison of transverse strains between field test and different designs at the bottom of the mid-span panel for load path 2](image)

Additionally, girder live load DFs for the proposed panel designs were estimated using vertical deflections of girders, and subsequently compared to those from the original design calculated with measured and estimated deflections using the FEM. The results from Table 6 indicate that DFs calculated for the different designs are fairly close to one another, as anticipated, since DF is mainly governed by the girders’ spacing.
Table 6- Comparison of Girders Live Load Moment DFs for the Different Designs

<table>
<thead>
<tr>
<th>Girder</th>
<th>Original design: Measured deflections</th>
<th>Original design: Estimated deflections</th>
<th>Redesign 1: Estimated deflections</th>
<th>Redesign 2: Estimated deflections</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interior</td>
<td>0.44</td>
<td>0.46</td>
<td>0.47</td>
<td>0.46</td>
</tr>
<tr>
<td>Exterior</td>
<td>0.34</td>
<td>0.31</td>
<td>0.30</td>
<td>0.31</td>
</tr>
</tbody>
</table>

To quantify the cost effectiveness of the proposed designs, the volume of the UHPC used in a single panel, then the bridge deck, are calculated and presented in Table 7. For this specific bridge, the UHPC volume is reduced by 8.8% and 13.4% for the first and the second redesigns, respectively. This reduction in volume would decrease UHPC material costs as well as associated labor expenses. Furthermore, reducing the number of joints would also provide additional cost savings.

The positive and negative moment demands at the strength-I limit state (AASHTO 2010) were also computed and compared to factored flexural resistance ($M_r$) of each panel redesign, in accordance with a design guide for UHPC waffle deck (Aaleti et al. 2013). The results indicated that each redesigned panel would provide adequate flexural resistance to satisfy strength-I limit state loading (see Table 8).

Table 7- UHPC Volume for the Different Designs

<table>
<thead>
<tr>
<th>Design</th>
<th>Single Panel Volume (m³)</th>
<th>Bridge Deck Volume (m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Original Design</td>
<td>1.61</td>
<td>22.54</td>
</tr>
<tr>
<td>Redesign 1</td>
<td>1.48</td>
<td>20.72</td>
</tr>
<tr>
<td>Redesign 2</td>
<td>1.42</td>
<td>19.88</td>
</tr>
</tbody>
</table>

Table 8- Strength I Limit State Moments for the Two Redesigns

<table>
<thead>
<tr>
<th>Redesign</th>
<th>Positive moment (kN-m/m)</th>
<th>Negative moment (kN-m/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Demand</td>
<td>$M_r$</td>
</tr>
<tr>
<td>1</td>
<td>43.5</td>
<td>49.9</td>
</tr>
<tr>
<td>2</td>
<td>43.4</td>
<td>49.9</td>
</tr>
</tbody>
</table>

According to the results of this finite element analysis, the two alternative designs can be used instead of the original design with acceptable structural performance. Evidently, the second
redesign is more economical than the first redesign. Nevertheless, proper experimental validation of the two recommended deck panel redesigns is recommended prior to implementation in practice.

Summary and Conclusions

A combination of field testing and an analytical study was conducted in this paper to assess the structural performance of the first bridge constructed with UHPC waffle deck panels. The field testing of the bridge included monitoring of vertical deflections and strains at discrete, critical locations on the bridge deck as it was subjected to static and dynamic truck loads. An FEM of the bridge was developed in order to construe the results of live load testing, estimate strains due to dead load, and to examine the live load moment distribution. Following the satisfactory structural performance of the bridge under live load testing, cost effective deck panel alternatives, to that implemented in the field, were then explored with the objective of minimizing the UHPC volume and associated labor and material costs. Using the FEM, the optimization of the waffle panels was undertaken by varying the number of ribs as well as spacing between ribs, such that the structural performance of the panels would not be compromised.

The following conclusions can be drawn from this study:

- The collected data for girder vertical deflections and panel strains indicated acceptable performance of the first bridge designed with UHPC waffle panels; none of the gages placed on the top of the deck registered strains close to cracking due to the application of live load.

- Only two strain gages at the bottom of the deck panels adjacent to the abutment did register strains greater than the expected cracking strain of the UHPC, due to preexisting cracks
observed prior to testing. Because these strains were not excessive and were located on the underside of the deck, no negative impacts to the performance and durability were expected for the waffle deck panels in this bridge.

- The maximum live load moment distribution factor for the interior girder was computed to be 0.51. This is considered acceptable due to this value being lower than the AASHTO recommended value of 0.66. In addition, the maximum dynamic amplification factor for the bridge girders was computed to be 1.4, which was close to the AASHTO recommended value of 1.33.

- For the first recommended optimized design, the number of transverse and longitudinal interior ribs, per panel, were effectively reduced from six to two and four to two, respectively. This design was found to be appropriate, which reduced the UHPC volume by 8.8% compared to the original design.

- The analyses showed that the longitudinal interior ribs could be completely removed without affecting the connectivity of two adjacent panels. Therefore, in the second recommended optimized design, all longitudinal interior ribs were removed while retaining only two interior transverse ribs per panel. This alternative was also shown to be effective, which reduced the UHPC volume by 13.4% compared to the original design, with potential additional saving, that resulted from a reduced labor cost.

- For both optimized deck panel designs, the live load moment distribution factors and strain distributions remained the same as those obtained for the original design.
References


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